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ENGINEERING PROPERTIES OF EXPANSIVE CLAYS

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ENGINEERING PROPERTIES OF EXPANSIVE CLAYS

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SYNOPSIS

Expansive clay soils have been encountered in rather widespread locations throughout the western United States at the sites of structures built or to be built by the Bureau of Reclamation. Inasmuch as most of the Bureau's structures are hydraulic, the normal difficulties encountered when expansive clays are present in the subgrade foundation are greatly magnified when these clays are completely saturated. As a result of some of the difficulties which have been experienced, a considerable amount of research testing has been done in order that expansive clays can be recognized and their potential swelling properties anticipated.

Such petrographic laboratory facilities as equipment for making microscopic examinations, X-ray diffraction determinations, and differential thermal analyses are quite valuable in determining the presence of objectionable clay minerals which may ultimately cause expansions. In addition to these more complex tests, relatively simple tests which can be performed in the average soil mechanics laboratory give excellent indices on expansion properties. Three such tests are those to determine colloid content, plasticity index, and shrinkage limit.

Numerous actual expansion tests have been performed in our laboratory to determine the actual volume change values for various types of clay soils and the uplift pressures which can be developed when the volume is maintained at a constant amount. The volume change data have been correlated with the three index values referred to above. In addition to these laboratory tests, the load-expansion and stability characteristics of a considerable number of soils have been studied. Methods of controlling expansion in remolded soils have been worked out by controlling placement moisture and density conditions.

Included in the paper is a discussion of several design and operation experiences on projects where difficulties were encountered or were anticipated in connection with the foundations of hydraulic structures. These include the uplift of the piers of the Malheur Siphon, the cracking and uplift of concrete canal linings in the Central Valley area of California, instability of clay slopes in this area, remedial measures inaugurated on later work in this area, the use of belled-out piles to resist

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uplift at the Wellton-Mohawk Pumping Plant No. 3, and the examination of bentonitic soils in the spillway subgrade of the proposed Tiber Dam.

INTRODUCTION

Expansive clay soils have been encountered in rather widespread locations throughout the west at the sites of structures built by the Bureau of Reclamation. The troubles encountered in connection with expansive soils when used for highway fills or subgrade foundations or when used for building foundations have frequently been described in literature. However, we believe this paper is one of the first which describes the difficulties encountered with such soils when used as foundation materials for a variety of hydraulic structures. It is quite obvious that the amount of uplift, the uplift pressures, and loss of stability can be magnified greatly in hydraulic structures, as compared with such nonhydraulic structures as buildings and roadways, because of the presence of water which will eventually saturate the foundation soils. The types of structures in which we are primarily interested are canals and appurtenant structures, pumping plants, power plants, and dams and appurtenant structures.

We first became actively interested in the destructive effects of expansive foundation soils in 1938 when considerable distress was noted in the piers and anchors of a large steel siphon across the Malheur River on the Owyhee Project in Oregon. Since that time other structures have shown distress from the destructive forces of expansive soils. A rather complete laboratory research program was initiated to secure basic data on these expansive soils so that their actions could be predicted and proper design and construction measures could be adopted to provide trouble-free structures.

In this paper we will describe the laboratory tests which were used to identify the expansive clay soils and study their engineering properties. In addition, some of the problems which developed when these soils were encountered will be discussed.

Laboratory Tests

Identification Tests

There are a number of laboratory tests which are useful in identifying clay soils and estimating potential swelling properties. Detailed characteristics of the clay minerals can be described by tests which are quite technical, such as microscopic examination, X-ray diffraction, and differential thermal analysis. To the practicing engineer, facilities for such tests may not be available and there is the need for more simplified test to identify expansive soil. Also, the engineer is interested in directly observing the effects of expansion for loadings and moisture conditions which simulate those in the actual structure.

The clay minerals of major importance to soils engineers fall within the montmorillonite, illite, and kaolinite groups. The montmorillonite clay minerals swell when wetted by water, whereas the clay minerals

of the other two groups do not swell or swell to a considerably less extent. The amount of swelling to be anticipated from a soil containing montmorillonite clay minerals is largely dependent upon the mount of this mineral which is present and the kind and amount of exchangeable bases. Where petrographic laboratory facilities are available, microscopic, X-ray diffraction, and differential thermal analysis techniques are very useful to determine whether expansions are likely. The tests are used to determine if expansive material is present in a soil, and, if so, the quantity, and other information about the exchangeable bases. Each of these methods have their particular use. All three methods are desirable in a detailed investigation of fine-grained material, as only by close coordination of the three methods can the mineralogical composition and influence of texture and structure by determined. Brief descriptions of these detailed tests are as follows:

a. The microscopic examination permits direct observation of an enlarged image of the material. An identification depends upon the fact that all materials reflect light to different degrees and that they possess somewhat definite optical properties which can be observed precisely in polarized light. Certain stains make possible easy identification of many clay minerals. The principal use is to determine mineralogic composition, texture, and internal structure. For fracture surfaces and fragments, the stereoscopic microscope is used, usually at magnifications from 10 to 200. This process permits examination of gross structures and textural relationships and identification of particles of gravel, coarse to medium sand, and organic remains. For examination in more detail and at higher magnification, thin sections are prepared. (Fragile soils require the impregnation of a suitable resin or plastic.) This process permits examination of any but the finest particles for identifying minerals and describing intimate details of internal structure and texture. It especially permits observations of the nature of voids and their inter-connection, grain-to-grain relationships, binding materials, and concretionary developments.
b. X-ray diffraction analysis supplements microscopic examination in identification of the finest fractions and in quantitative determination of the mineral constituents. The method depends upon the manner in which the atomic structure of the compound diffracts X-rays of a certain wave length and determines the arrangement of atoms within crystals. Amorphous substances, such as volcanic glass, opal, allophane, and cliachite are not detected by X-ray, and minor constituents, representing approximately 10 percent or less, cannot be observed. X-ray diffraction patterns may be recorded on photographic film. Film patterns permit comparison with standard reference patterns for identification of mineral constituents. This method may frequently yield misleading results where interlaying exists within individual crystals, such as when montmorillonite and kaolinite, or illite and montmorillonite alternate at random in the same crystal. For reasons such as this, precise analysis of most soils is a special and time-consuming

problem. Semiquantitative analysis, however, usually can be accomplished rapidly. In particular, connection with expansive clay, the change in dimensions of the lattice can be observed when patterns are made at varying humidity conditions.

c. The differential thermal analysis also supplements the microscopic examination in identification of the finest fractions and their relative abundance. The method involves the differential measurement of heat absorbed or evolved by a material while it is heated at a constant rate. It yields definite results only with crystalline substances, since only crystalline compounds exhibit several characteristic levels of energy release or absorption during heating in the usual ranges from room temperature to $1,100^{\circ}$ C. Many types of equipment have been developed, but few are available commercially. An international committee is now engaged in an effort to establish the requirements of a satisfactory instrument. This analysis is valuable because it reveals interlayering in clay minerals, whereas X-ray diffraction fails to do so. Also, distinction between clay minerals containing differing exchangeable ions is possible. A soil sample containing ingredients which have great energy exchanges might have the minor energy exchanges of other constituents masked. Observations by microscopy are especially valuable for this detection.

In addition to the more complex petrographic tests discussed above, there are other identification tests that may be performed in the average soil laboratory to determine possible expansive characteristics. The most simple of these is the free-swell test. This test is performed by slowly pouring 10 cc of dry soil passing the No. 40 sieve into a 100 cc graduate filled with water and noting the swelled volume of the soil after it comes to rest at the bottom. The free-swell value in percent is obtained by the expression, $\frac{\text{(final volume minus initial volume)}}{\text{initial volume}} \times 100.$
A good grade of high-swelling commercial bentonite will have a free-swell value of 1,200 to 2000 percent. Soils having free-swell values as low as 100 percent may exhibit considerable volume change when wetted under light loadings and should be viewed with caution, while soils having free-swell values below 50 percent very seldom exhibit appreciable volume changes, even under very light loadings.

It has not been possible to work out close correlations between free-swell values and actual total volume changes determined by controlled expansion tests conducted in laboratory consolidometers. However, general trends are apparent and, in many instances, the free-swell values will give sufficiently accurate data for preliminary design data. These trends are shown on Figure 1, which is a plot of free swell against the total expansion of undisturbed soil specimens placed in laboratory consolidometers and expanded from air-dry to saturated conditions. These consolidometer tests will be discussed in detail in subsequent paragraphs.

In searching further for simplified identifying tests, it was found that the colloid content of a soil and the Atterberg or consistency tests

provide good indicators of the expansive characteristics of clays when considered together. This is true for the following reasons:

a. The colloid content, obtained from the gradation test, indicates the amount of the colloidal-size fraction which is the most active part of any soil material contributing to expansion.
b. The plasticity index (PI), which demonstrates the magnitude of the range of moisture change possible while the soil retains a plastic condition, is related to expansion because the water in the voids and clay minerals consumes space, thus causing changes in moisture to reflect volume change. High plasticity indices, which are indicative of active soils, would, therefore, be necessary in order for a soil to change volume appreciably when going from a semisolid to liquid phase.
c. The shrinkage limit which describes, indirectly, the minimum volume to which a soil will shrink upon drying and is an expression of percent of water necessary to fill the void spaces when the soil is at minimum volume is valuable supplemental information to the other two data. A low-shrinkage limit would show that a soil could begin volume change at a low-moisture content.

DATA FOR MAKING ESTIMATES OF PROBABLE VOLUME CHANGES FOR EXPANSIVE SOILS FROM DELTA-MENDOTA AND FRIANT-KERN CANALS, CALIFORNIA

Data from index tests			Estimation of probable expansion* (% total volume change dry to saturated condition)	Indication of degree of expansion
Colloid content (% minus 0.001 mm)	Plasticity index	Shrinkage limit(%)		
>27	>32	<10	>30	Very high
18-37	23-45	6-12	20-30	High
12-27	12-34	8-18	10-20	Medium
<17	<20	>13	<10	Low

*Based on a vertical loading of 1.0 psi in the one-dimensional consolidation testing machine.

The above table is based on actual expansion tests for 38 undisturbed soil samples, and the results of these tests are summarized in Figure 2. The expansion tests from which the total volume change data were obtained are discussed in subsequent paragraphs. In Figure 2 the thin dashed lines were drawn on the basis of judgment so as to enclose most of the points and still keep the limits as narrow as possible. The values given in the table express the limits of these lines for the various degrees of expansion (low, medium, high, and very high) as compared to colloid content, plasticity index, and shrinkage limit. Since the data were obtained on undisturbed samples, with a variety of density conditions, the results of the measured total volume changes are somewhat irregular. More consistent and regular results could be expected from compacted specimens in which density conditions were controlled. However, our interest at the time of testing was primarily in identifying the expansiveness of undisturbed soils since they represented the actual

subgrade soils on which canal structures were to be placed. Although the initial or in-place density conditions varied, the range of variation in the Central Valley of California, where the samples were obtained, was not great. Therefore, the index data could be used to estimate expansiveness quickly in the field.

Other tests which were investigated for the purpose of estimating expansive characteristics included (1) size smaller than 0.005 mm; (2) liquid limit; (3) free swell; and (4) montmorillonite mineral content. However, from the standpoint of the soil mechanics engineer, it is believed that the three tests used provide a more simple and practical means of estimating the expansion characteristics.

Load-expansion Tests

The load-expansion test for providing factual data to predict the amounts of expansion is a form of the standard one-dimensional consolidation test. The testing equipment is similar to that used in most soil laboratories for consolidation studies. This equipment, called a consolidometer, is the fixed-ring type. It accomodates a remolded or undisturbed specimen 4-1/2 inches in diameter and 1-1/4 inches deep. Porous stones are provided at each end of the specimen for drainage or saturation. The specimen container is placed on the weighing table of a platform scale and the load is applied by a yoke actuated by a screw jack. The load imposed on the specimen is measured by the scale beam and a dial gage is provided to measure the amount of vertical movement (Figure 3).

The more-or-less standard procedure which we have adopted for testing the expansiveness of clays for concrete-lined canal subgrades requires two test specimens. These can be cut from undisturbed soil samples or remolded at desired moisture and density conditions from loose samples. After the specimens have been placed in the consolidometer ring and the original volume and moisture determined, they are allowed to air-dry to at least the shrinkage limit. The volume change at the air-dry condition is determined on the first specimen by a mercury displacement method. The second specimen is placed in the consolidometer, under a vertical loading of 1 psi (or any anticipated structure load) and saturated. The vertical movement indicates the volume change. From the above test we are able to determine the volume change and vertical movements from initial to air-dry moisture conditions and initial to saturated moisture conditions. The combined results for both specimens gives the total volume change for air-dry to saturated moisture conditions. The actual amount of volume change that occurred during typical tests is shown in Figure 3.

The 1 psi vertical loading was adopted early in our studies as the load to be applied while saturating the specimens because it represented the approximate loading on the subgrade of concrete canal linings which we were studying at that time. Because of the data accumulated under this load condition, the correlative data contained in this paper between index or identification properties and actual expansive characteristics are based on the total volume change from air-dry to saturated condition under the 1 psi loading.

Although the test just discussed was used primarily to indicate relative expansiveness of subgrade soils subjected to light loadings, other information can be secured by means of the one-dimensional consolidometer as job conditions require. In some instances third specimens are prepared and treated in the same manner as the second specimens discussed above, except that during the saturation period sufficient load was applied throughout the test to prevent vertical movement. By this process, the vertical uplift pressures were determined. In other cases it was desirable to make rather accurate predictions of the actual amount of movement that might occur under various load conditions. This was done by securing load-expansion data from the consolidometer test through varying the load conditions during the test. In these latter cases the sample was usually saturated directly from the initial placement condition while under a load to simulate field conditions.

As a matter of general interest, the maximum uplift pressure recorded for the expansive clays in the Central Valley of California was 147 psi. This pressure was that required tc hold the vertical height constant while the specimen was saturated from an air-dry condition in the consolidometer. In this test some lateral movement of particles took place as the cracks were being filled during expansion. It was felt that this test simulated the action that would occur under a structure placed on an air-dry expansive clay subgrade. Typical shrinkage of air-dry specimens is shown in Figure 4.

Some of the most interesting studies conducted involved the determination of the load-expansion characteristics so that actual uplift movements could be computed for various types of canal structures as well as the concrete canal linings. These involved undisturbed soil specimens where natural subgrade conditions were being studied and specimens remolded to anticipated field conditions where compacted subgrades and embankments of expansive soil were being studied. Figure 5 shows the results of some of the load-expansion tests conducted on undisturbed soil specimens from a pumping plant foundation in Arizona. These soils which were described as high-swelling clays were very nearly saturated (91 to 100 percent) in their natural state (initial test condition), although they were taken at an elevation many feet above ground-water level. It is interesting to note that, although these clays had high initial degrees of saturation, they expanded a great deal as free water was made available to them and as the load conditions were varied. It should also be noted that, if the specimens were first heavily loaded to permit no volume change during saturation, their expansions under reduced loadings of zero were not as great as for specimens which were wetted under zero (or similar light) loadings.

Figure 6 shows the difference in load-expansion characteristics for a soil which was loaded before saturation (Curve DE) and one that was saturated before loading (Curve ABC). Thus, it becomes apparent that the test procedure is quite important if accurate estimates of uplift are to be determined. If more accurate estimates of uplift are required on, for example, a thick layer of clay under a pumping plant, it is necessary to prepare several specimens for consolidometer tests. These specimens should then be loaded with the plant loading plus the weight of

overlying material from the base of the plant to the elevation of the material they represent. The specimens are then saturated under their respective loadings and the load-expansion data plotted from the uplift data obtained. We would then obtain a Curve BD which would be intermediate between the load-expansion curves obtained for saturation before loading and for saturation under an initial load large enough to inhibit expansion. In many studies it is not practical to go into such detail and sufficient data may be obtained by two tests to provide data as Curves ABC and DE; the Curve BD then can be drawn by judgment. Considerable judgment may be required, however, as the expansion may be very critical to loading as shown by Figure 7. In this case, very light loads inhibited the swell to a large extent. The shape of the DE curve in Figure 6, however, generally points up this fact. A few intermediate test points may be necessary for more detailed analysis of a BD curve.

A control of expansions can be obtained by decreasing the density of the soil to near its expanded density for the load conditions at which it will be saturated in a structure. Many times this can be accomplished in subsurface foundation soils by prewetting an area prior to construction. However, because of the time involved in saturating impervious clays of this nature, this method may not be practical and other design or construction measures must be adopted.

The control of expansion in compacted clay embankments or subgrades is considerably easier as density, and to some extent the moisture, can be controlled to give more favorable conditions. To study closely the effect of various moisture and density conditions on expansion, several series of tests were made in the laboratory consolidometers on soil specimens compacted at a variety of moistures and densities. For each placement condition duplicate specimens were prepared and tested to determine (1) the expansion under 1 psi loadings when saturated and (2) the uplift pressure or load required to restrain expansion (these specimens were then allowed to expand by reducing the restraining load in increments). Figures 8 and 9 show the results of these tests on "Porterville clays" from the Delta-Mendota Canal in the Central Valley of California for canal embankment studies. Figure 8 is a summary of the expansion data for specimens placed at approximately 15-, 20-, and 25-percent moisture contests and at densities of 77, 87, 92, and 97 pounds per cubic foot for each moisture condition. The placement conditions are indicated for each specimen by circular points on the figure. The amount of expansion in percent of initial volume is shown beside each point. The trend of expansion is shown by thin dotted lines drawn from these points to the triangular points which represent the final conditions of density and moisture content after expansion. The lines form a distinct fanning pattern, being steep for higher placement densities and flat for low-placement densities. A heavy dashed line has been drawn through the triangular points to represent the placement conditions for zero expansion for a 1 psi loading. A series of heavy dashed lines have been drawn to indicate placement conditions which would result in near equal expansions; each line is labeled to indicate the amount of expansion anticipated. It can be seen from studying this plot that a combination of placement densities which are lower than those obtained by

standard compaction and placement moistures near or higher than standard optimum are required to insure low amounts of expansion. It is evident that the reduction of density alone will not always provide the desired reduction in expansion for light loadings. Increased moisture provides good expansion control, especially in the higher expansion ranges; however, it is usually not possible to place these soils at high moisture contents because of construction difficulties. Therefore, a combination of moderately high moisture and low density control is most suitable.

Figure 9 is also of interest in studying the effect of various placement densities and moistures on uplift pressures, where the initial volume was held constant during the consolidometer test. In this figure, the amount of load (psi) necessary to hold the volume at no expansion is shown alongside each point of placement density and moisture. On the basis of these values, the heavy dashed lines were drawn to represent placement conditions for which equal values of total uplift pressure will result from wetting. In this case, it can be seen that considerable advantage is gained by reducing density without necessarily increasing the moisture content.

By performing numerous load-swell tests, it was found that the remolded clays behaved much as the undisturbed clays. A specimen which is loaded to a very light load (as the 1 psi load) expanded considerably more than an identical specimen which was first loaded sufficiently to prevent expansion and then allowed to expand by reducing the load in increments. Figure 10 was drawn to show the expansion trends under light and heavy loadings. The heavy lines, AD and ABC, represent the example of expansive characteristics of specimens placed at a certain moisture-density placement condition. When one specimen was loaded to only 1 psi, it expanded according to line AD when saturated. When a second similar specimen was loaded to prevent expansion, it moved along horizontal line AB when saturated. When the loading was reduced to 1 psi, it expanded along line BC but never expanded to the amount of the first specimen by the density difference between C and D. Similar information, plotted as the load versus expansion, was previously shown for undisturbed soils in Figures 5 and 6.

Stability Tests

In addition to the difficulties which can be encountered from the uplift of expansive clays when saturated, the loss of stability or shear strength is an important factor to be considered. Generally speaking, the shearing strength becomes less as the moisture contents are increased and the densities are decreased when the clays are expanded through saturation. This loss of strength may assume large proportions. Our laboratory has not conducted basic research on this phase with expansive clays but we have considerable data collected in the course of regular project work. The extreme loss of strength that may occur in an undisturbed expansive clay is well shown by average test results obtained on four specimens from a sample of very fat clay[3] (CH) secured

3. Average LL = 75, PI = 50, SL = 12, colloids = 59 percent.

from the foundation area of Pumping Plant No. 3 of the Wellton-Mohawk Canal System, Arizona. This clay, which had a dry density of 103 pcf and was 93 percent saturated in its natural state, had an unconfined compressive strength of 155 psi. When saturated, under no loading, the material expanded to an average dry density of 85.7 pcf at 99-percent saturation, and the unconfined compressive strength was reduced to practically zero.

The strength loss of compacted (remolded) Porterville clays from the Central Valley of California is illustrated by tests made on canal lining clays from the Friant-Kern Canal System. At one location the clay,[4] which was compacted to maximum laboratory dry density of 94 pcf and was 70 percent saturated at optimum moisture (22 percent), had an unconfined compressive strength of 28 psi. Upon saturation under a low loading, the density decreased to 91 pcf, the degree of saturation increased to 94 percent, and the unconfined compressive strength was reduced to 4 psi (data based on averages of four specimens). Thus, it can be seen, from this last data, that although the amount of density change may be small, the strength can be materially reduced as a result of wetting.

If these remolded clays are air-dried prior to saturation, further reductions in strength can be anticipated. For example, one set of specimens which were remolded at maximum laboratory density and optimum moisture showed a strength loss to 4 psi when saturated from the optimum moisture condition and to 1 psi when saturated after air-drying. Numerous tests substantiate the trends of the examples discussed above and further discussions are given in subsequent paragraphs on field operations.

Design and Operation Experiences

The following paragraphs include some general discussions on several structures in which soil expansions have been detrimental to structures in operation or were considered dangerous and the designs were made to avoid subsequent trouble.

Malheur River Siphon, Oregon

The upward movement of the piers of the Malheur River Siphon, located in the eastern part of Oregon, furnishes an interesting example of structural difficulties that can occur from foundation expansion.[5] This siphon consists of an 80-inch steel welded pipe 21,976 feet long and a 78-inch precast reinforced concrete pipe 1,020 feet long at the inlet and outlet ends. The steel pipe lies entirely above ground and is supported upon piers and anchors at maximum spans of 60 feet and 972 feet, respectively. The pipe is supported at each pier by two steel rockers

4. Average LL = 57, PI = 37, Class = CH.
5. "Foundation Displacements Along the Malheur River Siphon as Affected by Swelling Shales," by R. C. Mielenz and C. J. Okeson; Economic Geology, Volume XLI, No. 3, May 1946.

which are set against bearing shoes on the pipe. One expansion joint was placed in the pipe between each pair of anchors to allow movements accompanying load and temperature changes. In the Spring of 1938, within 3 years after construction, significant distress was noted at supports in the section of the siphon lying north of the river. This section represented about 10 percent of the total siphon length. Five surveys were made at intervals between 1938 and 1943. At certain piers the displacements were greater than the displacements for which the rockers were designed, the maximum being 0.45 foot. Maximum pier movements of 0.73 foot longitudinally, 0.58 foot laterally, and 1.07 feet upward were recorded. A review of the five surveys indicated that the uplift was progressive but at a declining rate.

A foundation investigation program was undertaken in 1943 so that remedial measures could be initiated. This program consisted of boring, sampling, and laboratory testing. The bedrock in the general vicinity is the Idaho formation and in the immediate vicinity is composed predominantly of shale. The material directly underlying the siphon consisted of sand and sandy silts, clayey silts, and lean to fat clays. The clayey soil represented weathered shale and claystone of the formation. Laboratory tests showed that the silty and sandy soils were nonbentonitic and nonexpansive, the lean clays slightly bentonitic and slightly expansive under very light loads, and the fat clays were highly bentonitic (high montmorillonite content) and moderately expansive (over 10 percent under zero loading, 7.0 percent under a 5 psi loading, and 0.1 percent under a 20 psi loading, when saturated). The effect of expansion on the siphon varied with the natural water content of the soil, the depth of nonexpansive overburden soils, and character of the fat clays. It was found through the exploration work that the cause of expansion after construction was due to wetting the expansive clays primarily by seepage waters from the canal above the siphon. However, some addition to the ground water was derived from nearby irrigation and precipitation. The measures to prevent further displacements included (1) lining 2,000 feet of the canal beyond the outlet of the siphon and 1,000 feet of a nearby lateral; (2) placing a compacted earth blanket and providing drainage gutters along critical parts of the siphon to prevent wetting from precipitation; and (3) the removal of certain agricultural lands from irrigation. Figure 11 shows the distress in the rocker arms at the piers.

Friant-Kern Canal

Another interesting example involving uplift from expansive clays is the heaving of concrete canal linings on the Friant-Kern Canal, Central Valley Project, California. The clays in this area, which are locally known as "Porterville Clays," contain appreciable proportions of the montmorillonite mineral, are very fine grained, and are very plastic when wetted. Some of the physical properties of these clays were discussed in previous paragraphs. In the construction of the Friant-Kern Canal, the expansive clays were encountered in several reaches along the canal and to varying depths below the surface. For economic reasons it was necessary to construct the fill section out of the clay soils

existing along the canal line. Considerable distress has occurred at several locations along this canal when highly expansive subgrade clays were saturated by water seeping through joints and cracks in the lining. Distress first took place in the bottom of the canal where cracking of the lining at the toe of the slopes and heaving at the joints occurred, Figure 12. It has been found that expansions on the side slopes are not generally as critical as expansion of clays existing as bottom subgrade materials because the slope linings are less restrained than bottom linings and can move to some extent. However, there have been instances where some side heaving has occurred, necessitating replacement of lining panels (Figure 13). Failures of this nature are considered to be due to uplift forces rather than to slope instability.

There have also been instances at Delta-Mendota Canal, California, where longitudinal cracks up to 1-1/2 inches wide have opened in the slope lining. Similar Porterville clays were used on this project to construct embankments on partial fill sections. The cracks are apparently caused by an uplift of the upper portions of the embankment which heave upward as they become saturated and drag the upper portion of the lining with it. As the tensile strength of this unreinforced concrete is exceeded, cracking and movement take place. On this project the clays were only very slightly expansive at grade depths and no bottom cracking has occurred. No side cracking from movements normal to the slope have occurred as the slope lining appears to be sufficiently flexible to move with the soil in this direction. A large-scale field test was made during construction to test the effect of expansiveness on the slope linings of this project. Highly expansive clays used in the embankment were saturated by artificial means and elaborate measurements of all movements were taken. Although considerable lateral movement of the slope lining, varying from zero at the toe of the slope to about 0.11 foot at the top, took place, no cracking occurred; thus it was concluded that the slope lining was flexible enough to withstand anticipated movements. This has generally been true.

Remedial measures contemplated for later construction on the Friant-Kern Canal consisted of (1) prewetting the subgrade soils prior to final trimming and lining; (2) the use of asphalt membranes under the concrete lining; (3) the use of a heavy earth lining (3-foot normal thickness) instead of concrete; and (4) controlled embankment placing procedures. Difficulties were encountered when prewetting was attempted because the clays became so plastic and sticky that the trimming machine could not handle the soil. An asphalt membrane was placed on the subgrade for one reach of canal prior to lining construction. This appears to have reduced major cracking. Some troubles have been had with earth slides on the earth-lined sections (Figure 14). This has been due to extreme cracking of the earth lining prior to introduction of water into the canal and the loss of strength from expansion of natural foundation soils upon saturation. It is probable that the use of earth linings constructed of select nonexpansive soils would provide the most suitable lining where linings are considered necessary in highly developed agricultural areas such as these, if such select soils can be obtained within a reasonable haul distance. The fourth plan was used in the construction of equalizing

reservoir embankments. For this plan the materials were placed at lower densities and higher moisture contents than those specified for normal soils. These embankments were placed as wet as conventional compaction equipment would allow and at densities which, as indicated by laboratory tests, would provide minimum expansions when wetted and minimum consolidations if required to carry the water load prior to saturation. Thus, a minimum total movement was anticipated for the concrete lining.

Tiber Dam Spillway

Field explorations showed that the foundation of the spillway site for the proposed Tiber Dam, near Chester, Montana, contained bentonitic layers or seams within the firm Colorado shale bedrock. A testing program was carried out to study the consolidation or expansion and stability properties of these bentonitic soils.[6] The problems which were of concern were (1) uplift from expansion when saturated under light loads; (2) settlement of the layers when under heavy loads of the dam; and (3) the shearing strength of the saturated clays. Load-consolidation and triaxial shear tests were made on several samples to answer these questions. Although the bentonitic soils were practically saturated in this natural state, it was found that appreciable expansion or uplift would occur when water was supplied to the soils under loads less than 10 to 15 psi. Consolidations of up to 13 percent were obtained under loadings of 220 psi. Shear values were difficult to obtain because of nonuniformity of specimens but apparent values of cohesion = 9 psi and tan ϕ = 0.2 were indicated. The results of the expansion tests point out that removal of materials to 10 or 15 deep and replacement with nonexpansive soils would be required in critical areas of the spillway to provide assurance against uplift.

Wellton-Mohawk Pumping Plants

The investigations of the foundations of the Wellton-Mohawk Pumping Plants, near Yuma, Arizona, showed that a series of alternating strata of fat clay, silty clay, and silt existed at the proposed sites of Plants 2 and 3. It was found that the strata of fat clay were highly expansive. At Pumping Plant No. 2 the ground-water table was above foundation grade. Inasmuch as the expansive clays were saturated and the difference in load between the overburden at grade and the plant loadings were small, no uplift from the clays was anticipated. At the Plant No. 3 site no ground water was encountered in holes extending to 35 feet below the foundation grade, and tests showed the soils to be expansive under loadings representing the plant loading plus the weight of the soil to a depth of about 20 feet below the grade line. To provide protection against uplift (particularly differential uplift) 25-inch-diameter reinforced concrete piles 20 feet long were specified. Holes for the piles were drilled

6. LL = 112 to 254, PI = 78 to 216, SL = 4 to 15, colloid content = 31 to 54 percent, montmorillonite mineral = 45 to 90 percent.

and the bottom of the holes were belled out to provide anchors. It was felt that the 20-foot length placed the pile anchors in a relative inactive zone even after the foundation became saturated from the canal waters. Water was in this canal system for the first time in the Summer of 1951. Measurement points have been provided at each of the three plants so that records of settlement or uplift can be made. To date, no uplift has been reported.

CONCLUSIONS

It has been found that expansive clay soils can produce detrimental uplifts to hydraulic structures placed on them. The amount of expansiveness is dependent upon the amount of montmorillonite mineral present in the soil and the kind and amount of exchangeable bases.

Petrographic Laboratory tests are useful in identifying expansive soils and in determining expansiveness by determining the amount and type of minerals present. Petrographic microscope, X-ray diffraction, and differential thermal analysis techniques are used for this purpose.

Common soil laboratory tests as the colloid content, plasticity index, and shrinkage limit tests are also effective in determining the expansiveness of soils. A simple free-swell test provides a quick qualitative measure of expansiveness.

In contrast to the index tests above for estimating potential expansiveness, the soil laboratory consolidometer may be used to obtain quantitative uplift values for anticipated load conditions. These tests are usually made by saturating undisturbed or remolded subgrade soil specimens under the load conditions anticipated for the structure. Load-expansion curves may thus be developed for a variety of load conditions. The expansions obtained on similar soil specimens which are allowed to saturate under variable load conditions (zero to maximum uplift) and then brought to a common load condition are generally considerably different. Therefore, it is important to duplicate the anticipated field conditions of saturation and loading as closely as possible.

Since the volume change of a particular expansive clay soil when saturated under a specified loading is dependent upon the moisture content and density at the time saturation is begun, it is possible to control the amount of expansion in compacted clay soils by increasing the moisture and/or decreasing the density. The initial density and moisture conditions for expansion control can be worked out from laboratory consolidometer tests.

The increased moisture and decreased density resulting from the expansion of montmorillonite clay soils have a tendency to reduce the shearing strength of these soils. Under very light load conditions the cohesion of the soils can be reduced to practically zero upon saturation, even though the soils may have fairly high cohesive strengths at moderate moisture conditions.

When hydraulic structures are to be constructed in areas where expansive clays exist, it is necessary to take special precautions against uplift unless the structure loads are sufficient to resist the uplift forces. Stability should also be checked. Where structure loads are not

sufficient to prevent expansion of the subgrade soils, the following measures may be adopted:

a. The natural subgrade expansive clays may be removed and replaced with nonexpansive soils to a depth sufficient to provide loadings that will resist uplift.
b. When the subgrade is a remolded expansive soil, the volume change may be controlled by compacting the soil at high moisture contents and low densities as predetermined laboratory tests unless large periodic changes in moisture are anticipated. It may be necessary to check for consolidations during any presaturation period, however.
c. The saturation of the foundation soils may be prevented by cutting off all sources of water supply, although this may be difficult and impossible in come cases.
d. Piles anchored in an inactive zone below a structure provide good assurance against uplift if properly incorporated into the structure base.

It may be possible to control the expansions of montmorillonite clays by changing their exchangeable bases, as changing a sodium clay to a calcium clay. This, however, has not been attempted by us in actual construction practice.

The Soils Laboratory can be of assistance to designers in identifying clay soils which may produce detrimental uplifts to their structures. By means of proper field explorations and laboratory tests any dangers can be recognized, quantitatives uplift values determined, and recommendations for protective measures can be made.

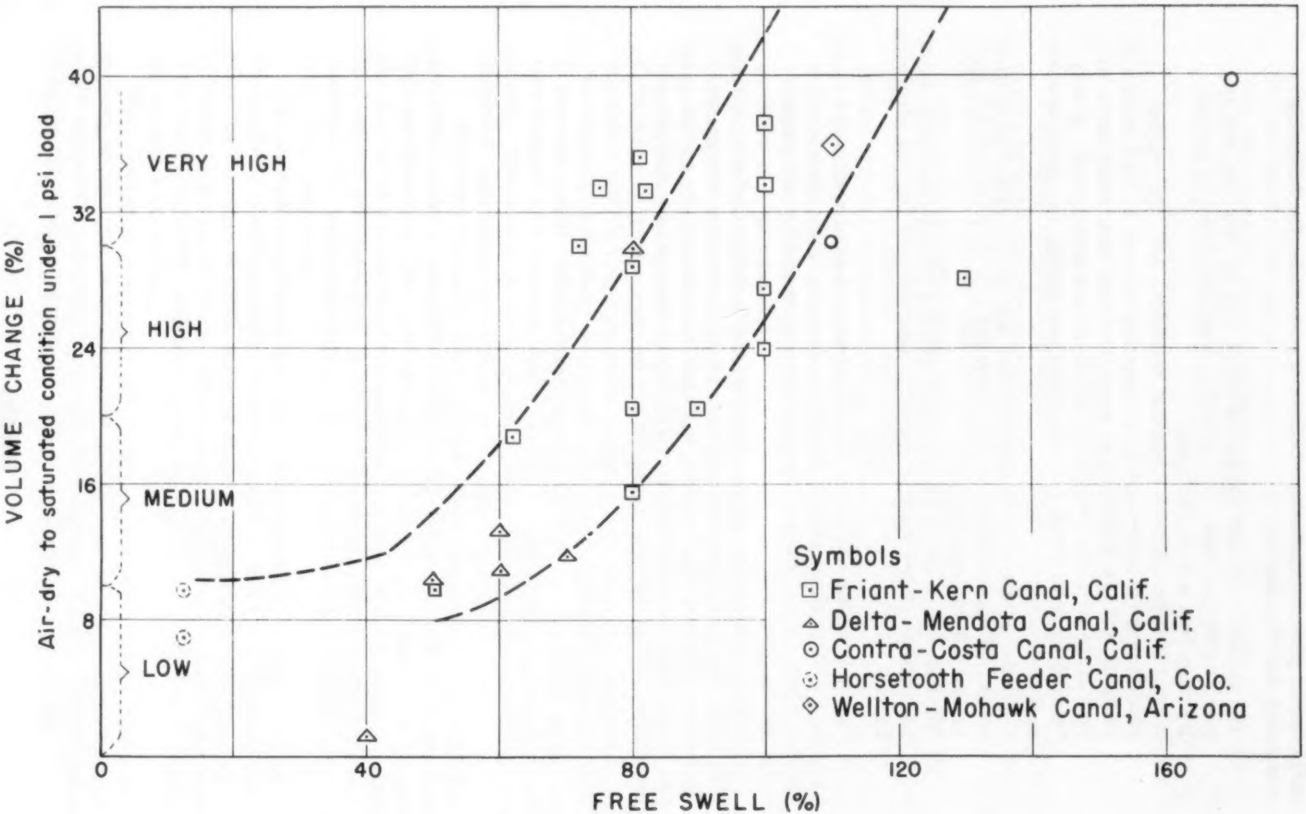


Figure 1 - FREE SWELL VERSUS VOLUME CHANGE

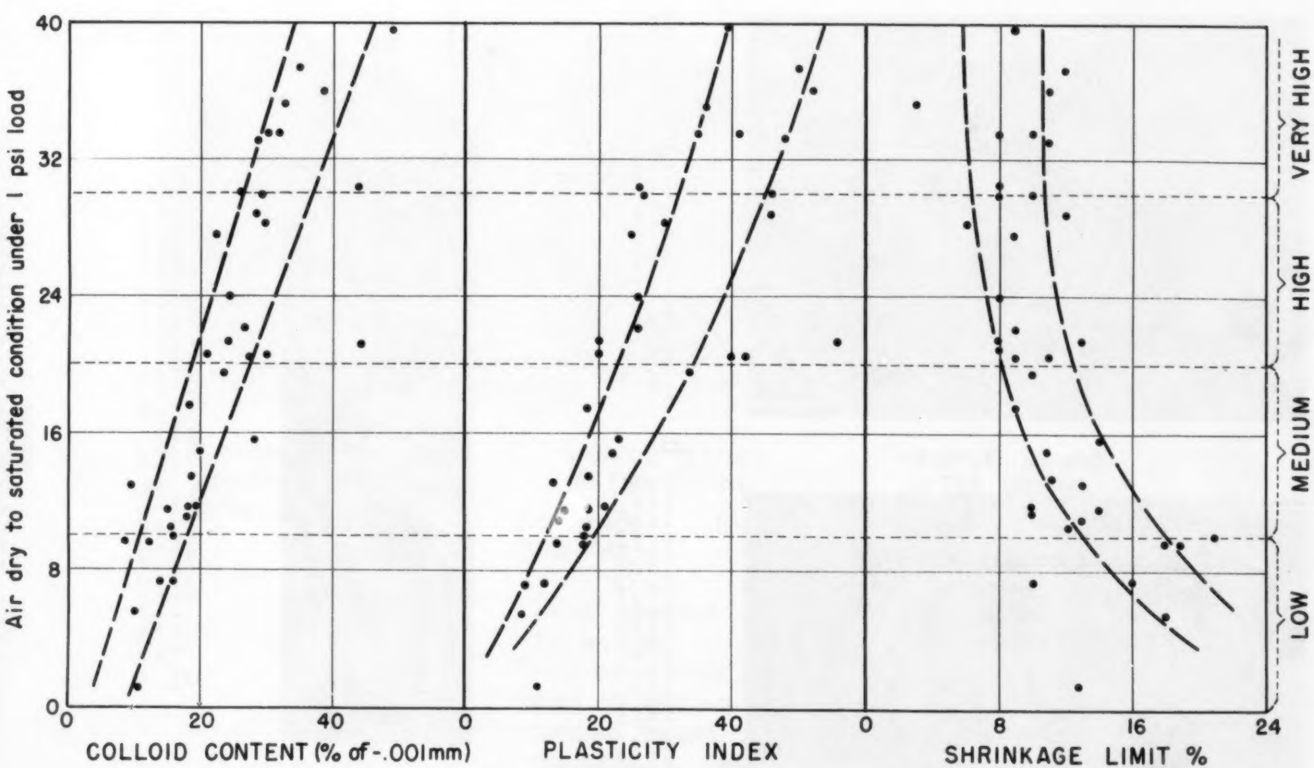
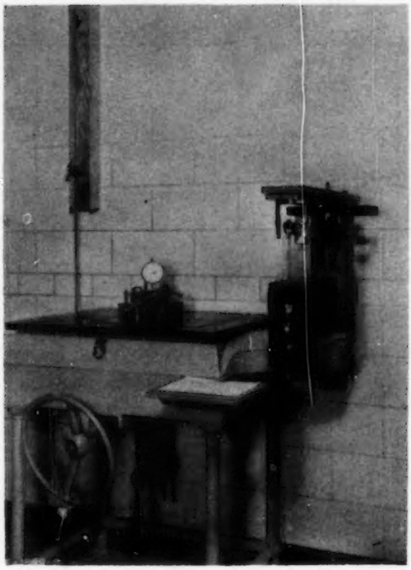
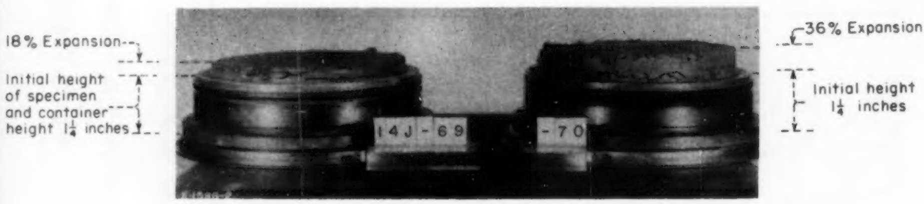


Figure 2 - RELATION OF VOLUME CHANGE TO COLLOID CONTENT, PLASTICITY INDEX, AND SHRINKAGE LIMIT

a. ONE OF THE CONSOLIDOMETER TESTING MACHINES WHICH ARE USED FOR VOLUME CHANGE MEASUREMENTS DURING LOADING. WATER MAY BE ADDED TO OBSERVE EFFECT OF WETTING ON VOLUME CHANGE



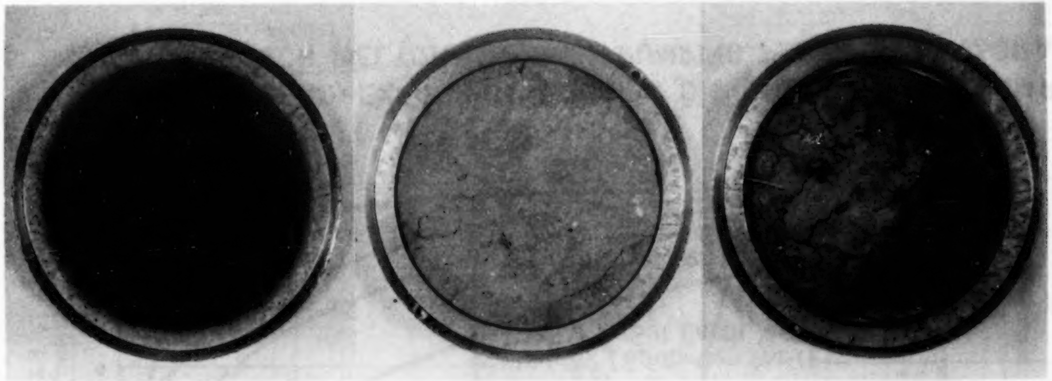
b. TEST SPECIMENS ARE CUT FROM UNDISTURBED SAMPLES AND PLACED IN THE TESTING CONTAINER WITH EXTREME PRECISION



C. TEST SPECIMENS OF EXPANSIVE CLAY SHOWING THE VERY HIGH VOLUME CHANGES WHICH WERE CAUSED BY WETTING THE SOILS WHILE THEY WERE UNDER A LOADING EQUIVALENT TO THAT OF A CANAL LINING

FIGURE 3

LABORATORY STUDIES OF EXPANSIVE CLAYS



Specimen at in-place condition immediately after cutting

SAMPLE 3T-102
AFTER DRYING

Shrinkage_ _ _ _ 8.4 %
Other data:
Swell above initial condition by wetting_ _ _ _ 19.3 %
Total vol. change_ 27.7%
Uplift by wetting while holding height constant_ _ _ _ 146.6 psi

SAMPLE 3T-103
AFTER DRYING

Shrinkage_ _ _ _ 16.4%
Other data:
Swell above initial condition by wetting_ _ _ _ _ 7.6%
Total vol. change 24.0 %
Uplift by wetting while holding height constant_ _ _ _ _ _ 11.1 psi

Figure 4 SHRINKAGE OF EXPANSIVE CLAY BY DRYING FROM IN-PLACE CONDITION

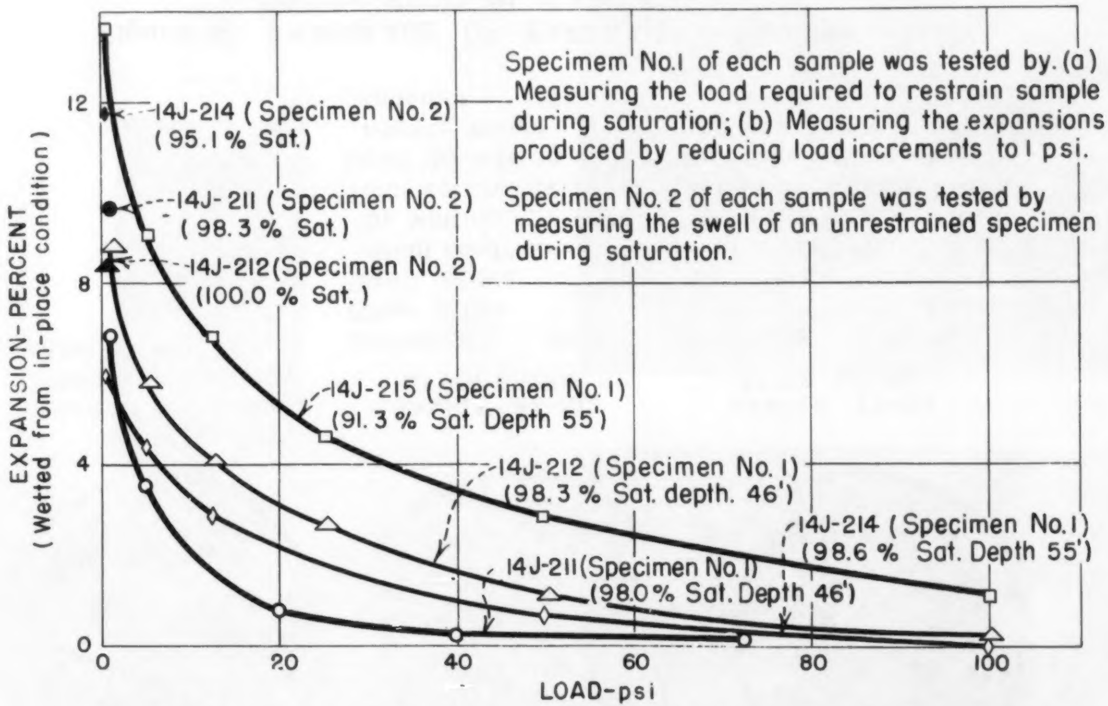


Figure 5. LOAD-EXPANSION STUDIES ON UNDISTURBED SAMPLES FOR WELTON-MOHAWK PUMPING PLANT IN ARIZONA

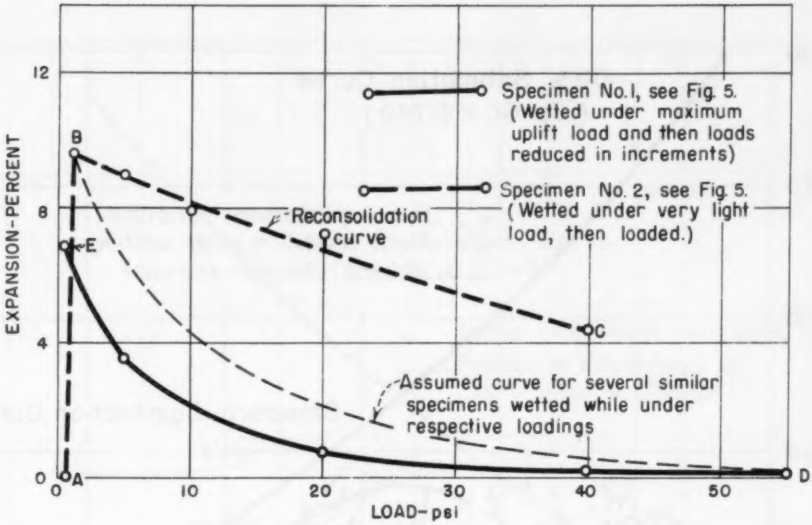


Figure 6. EFFECT OF SEQUENCE OF LOADING AND WETTING ON EXPANSION

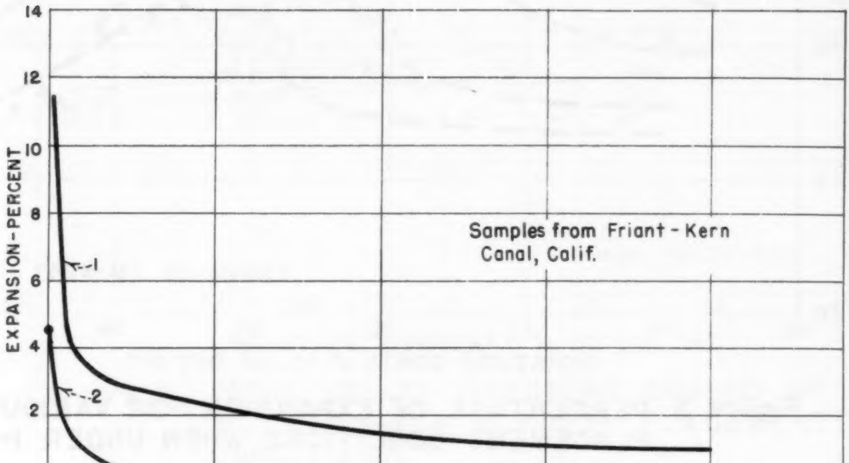


Figure 7. LOAD-EXPANSION DATA WHERE SOILS ARE CRITICAL WITH RESPECT TO LOADING

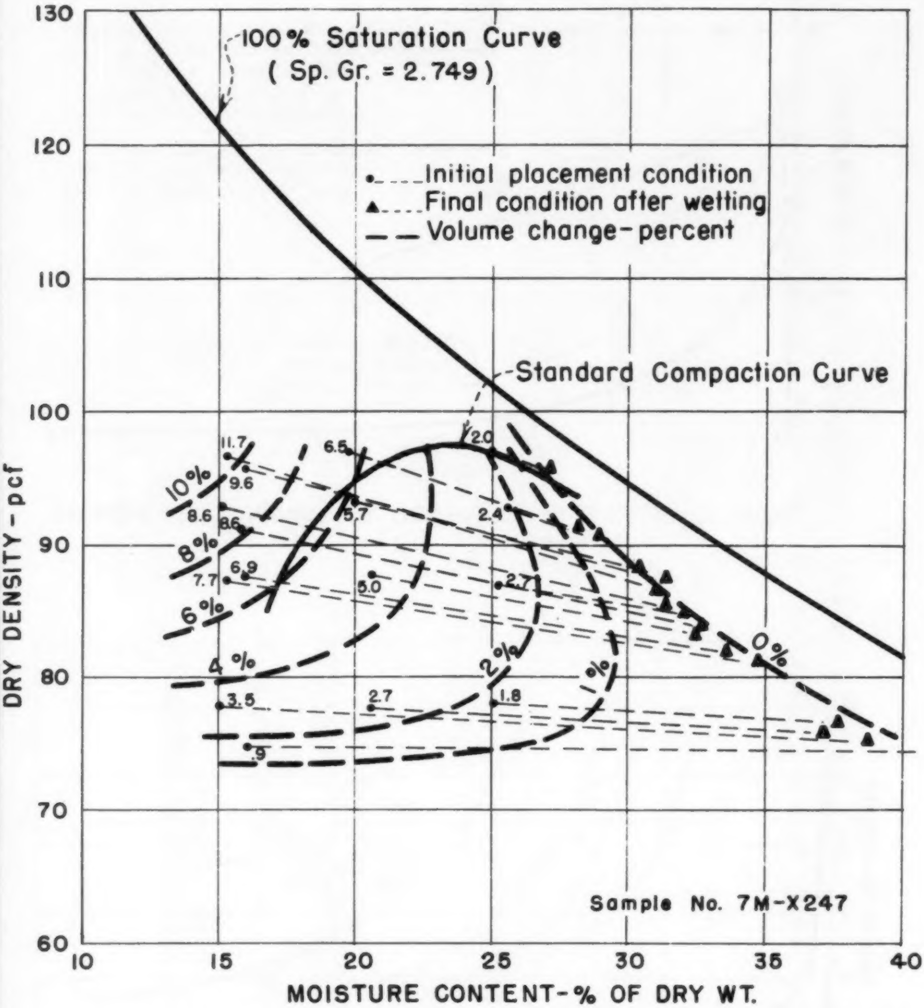


Figure 8. PERCENTAGE OF EXPANSION FOR VARIOUS PLACEMENT CONDITIONS WHEN UNDER 1-PSI LOAD

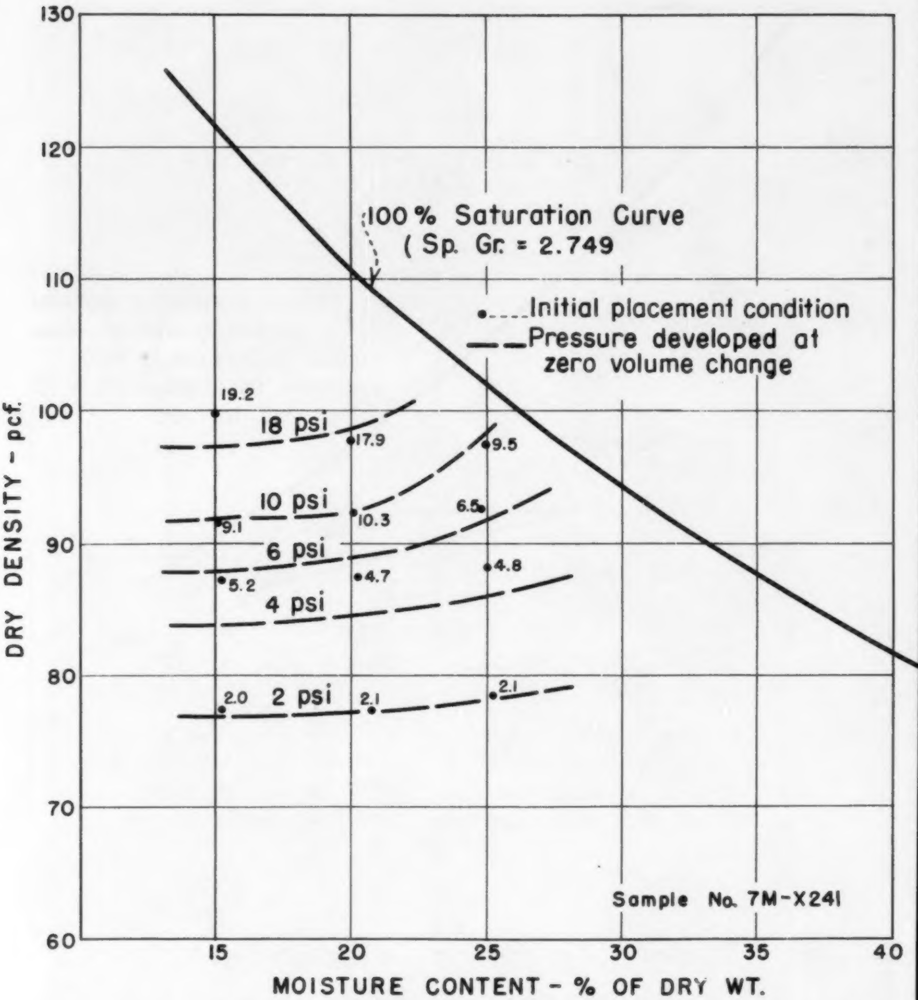


Figure 9. TOTAL UPLIFT PRESSURE CAUSED BY WETTING—FOR VARIOUS PLACEMENT CONDITIONS

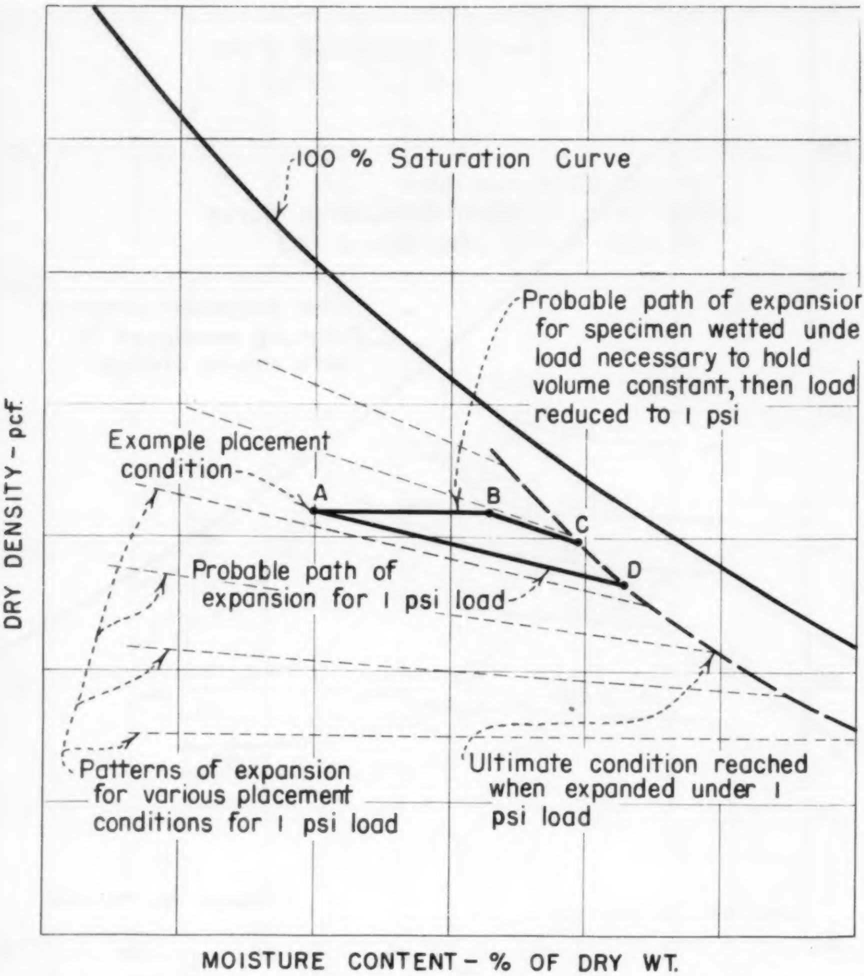


Figure 10. DEMONSTRATION OF EXPANSION TRENDS WITH RESPECT TO SEQUENCE OF LOADING AND SATURATING

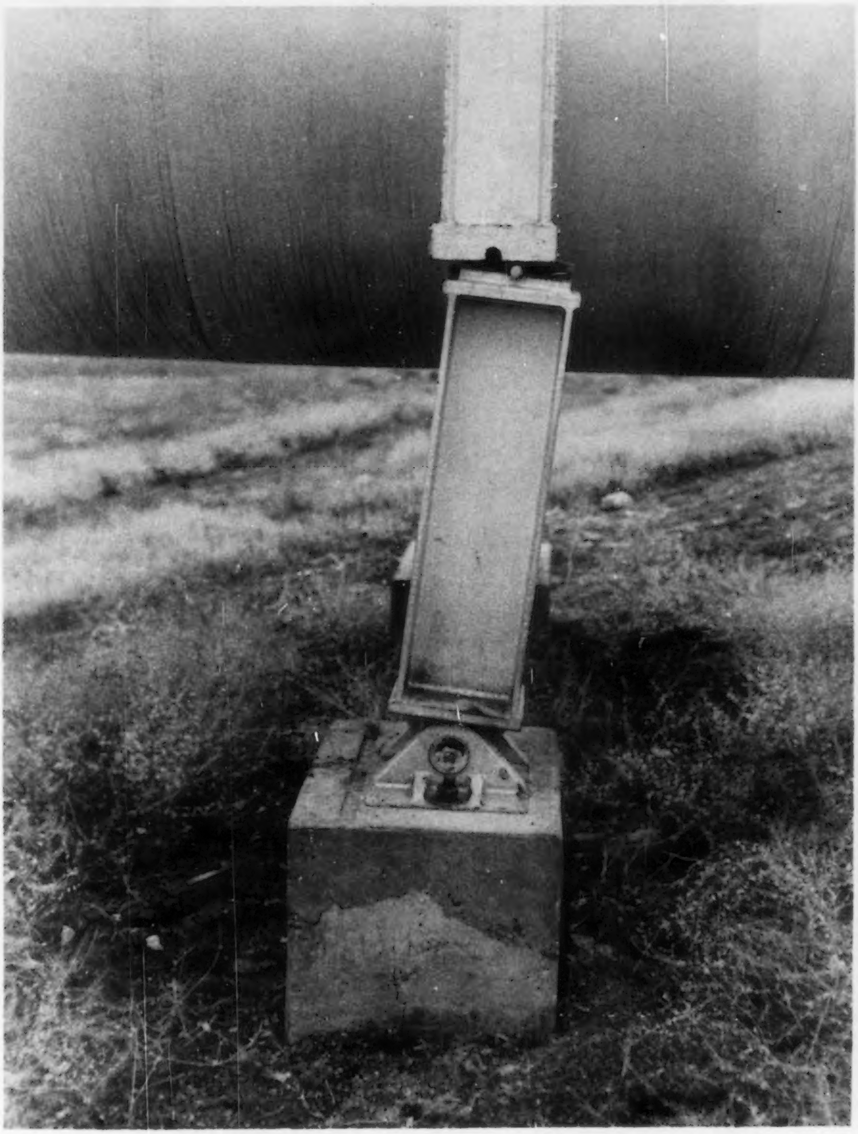
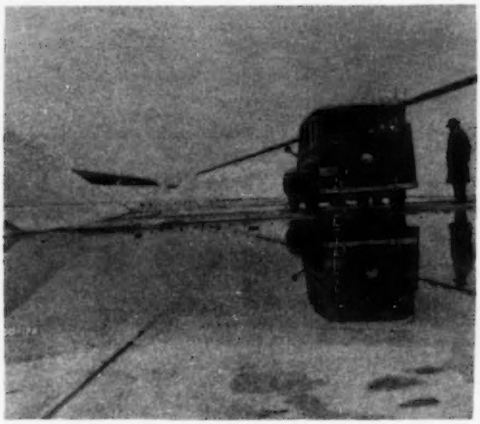


Figure 11. DISTRESS IN ROCKER SUPPORTS AT PIERS OF THE MALHEUR SIPHON, OREGON

a. CRACKING OF CONCRETE CANAL LINING AS A RESULT OF EXPANSIVE FORCES OF SWELLING CLAYS WHICH HAVE BEEN WETTED.



b. GENERAL HEAVING AT JOINTS CAN BE SEEN OVER CONSIDERABLE REACH OF THE CANAL. DRY AREAS ALONG TRANSVERSE JOINTS SHOW UP THE HEAVING AT THE JOINTS

FIGURE 12. EFFECTS OF SEEPAGE INTO THE SUBGRADE OF EXPANSIVE CLAY, FRIANT-KERN CANAL, CALIFORNIA

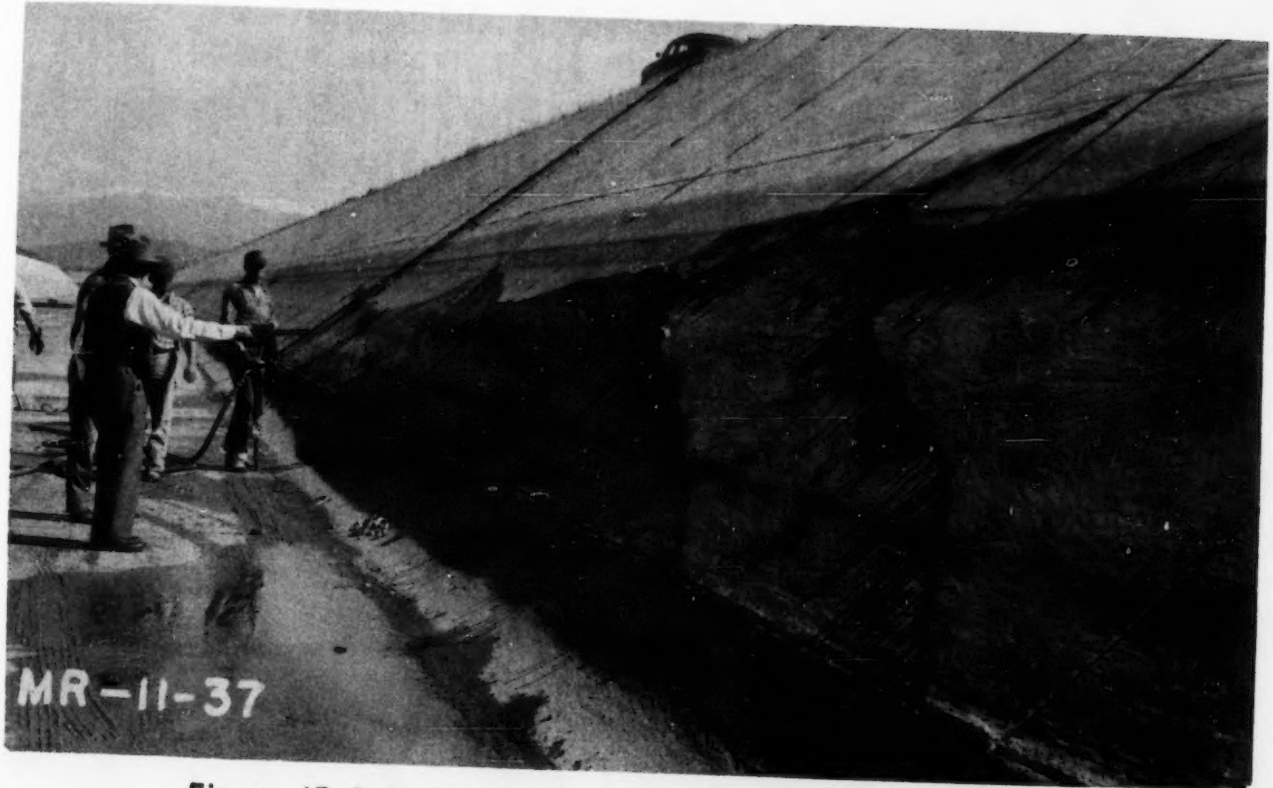


Figure 13-DISRUPTING OF CONCRETE CANAL LINING BY EXPANSION OF SUBGRADE CLAYS.

Figure 14. TYPICAL SLIDE IN COMPACTED EARTH LINING OF EXPANSIVE CLAY SOIL

PROCEEDINGS-SEPARATES

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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DECEMBER: 359(AT), 360(SM), 361(HY), 362(HY), 363(SM), 364(HY), 365(HY), 366(HY), 367(SU)[c], 368(WW)[c], 369(IR), 370(AT)[c], 371(SM)[c], 372(CO)[c], 373(ST)[c], 374(EM)[c], 375(EM), 376(EM), 377(SA)[c], 378(PO)[c].

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APRIL: 428(HY)[c], 429(EM)[c], 430(ST), 431(HY), 432(HY), 433(HY), 434(ST).

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JUNE: 444(SM)[e], 445(SM)[e], 446(ST)[e], 447(ST)[e], 448(ST)[e], 449(ST)[e], 450(ST)[e], 451(ST)[e], 452(SA)[e], 453(SA)[e], 454(SA)[e], 455(SA)[e], 456(SM)[e].

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a. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
b. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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